KAAR & KETTRON

Test of a Highway Bridge

Civil Engineering

B. S.

1911

UNIV. OF
ILLINOIS
LIBRARY

UNIVERSITY OF ILLINOIS
LIBRARY

Class 1911
Book K11
Volume

1022
73

TEST OF A HIGHWAY BRIDGE

BY

HOWARD WILLIAM KAAR
AND
HENRY PEARSON KETTRON

THESIS

FOR THE

DEGREE OF

BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

IN THE

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

1911

1911
K11

UNIVERSITY OF ILLINOIS

May 25, 1911

I recommend that the thesis prepared under my supervision by GUY ATCHISON KAAR and HENRY PEARSON KETTRON entitled Test of a Highway Bridge be approved as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

F. O. Dufour
Asst. Professor of Structural Engineering.

Recommendation approved:

Ira O. Baker.
Head of the Department of Civil Engineering.

197657

CONTENTS.

-----------0----------

Digitized by the Internet Archive
in 2013

http://archive.org/details/testofhighwaybri00kaar

I. INTRODUCTION.

---------0---------

In recent years many attempts have been made to establish a formula for computing the stresses in the members of a bridge due to the passage of live loads. In the case of a load moving over a single beam, a reasonably accurate formula may be deduced. In the case of a more complicated system of members such as is found in a bridge, the problem has very great difficulties. The stresses depend upon the speed and weight of the moving load, and to a small extent upon the centrifugal force introduced by the deflection of the bridge.

The formulae which are in general use are of two classes. One class takes into consideration the length of span loaded, by involving the ratio of the static live load stress to the length of the bridge which is loaded. The most common form for the formula of this class is

$$I = S\left(\frac{a}{L + b}\right)$$

Where

I = impact stress to be added to the static live load stress.

S = the static live load stress.

L = the length in feet of the portion of the bridge that is loaded to produce the maximum stress in the member, and

a and b = constants expressed in feet.

There are many different values given for these constants. For use with highway bridges, J. A. L. Waddell specifies a = 100 feet and b = 150 feet, and Milo S. Ketchum specifies a = 150 feet and b = 300 feet. The American Bridge Company does not use a formula for highway bridges but requires the maximum stress to be increased by 25 per cent.

The other class of formulae considers the impact stress to be dependent upon the ratio of the live load stress to the dead load stress. This class is represented by the formula

$$I = S\left(\frac{S}{S + D}\right)$$

Where

I = impact stress to be added to the static live load stress.

S = the static live load stress, and

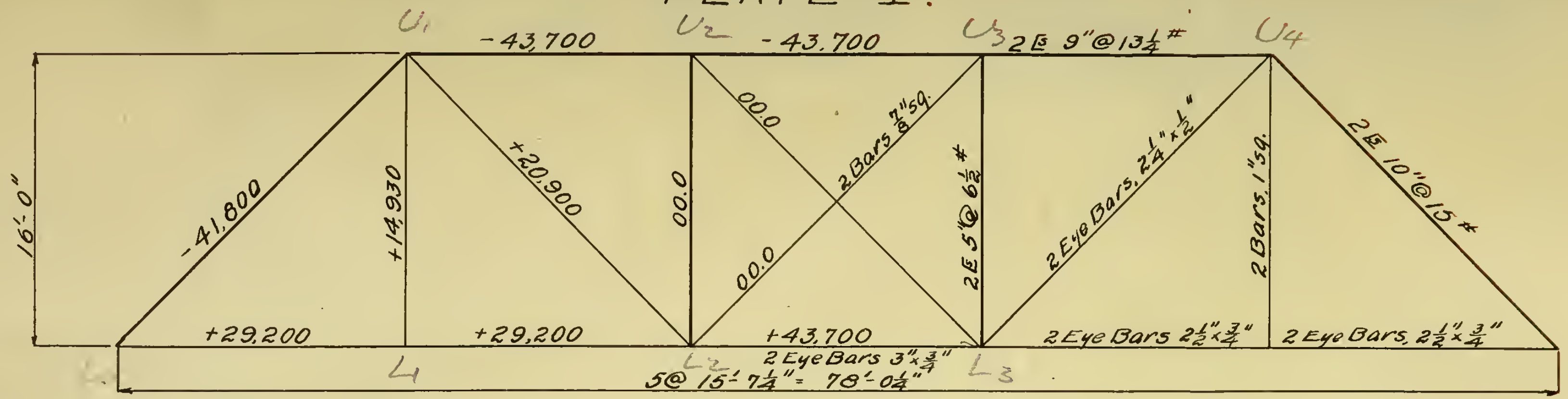
D = the dead load stress.

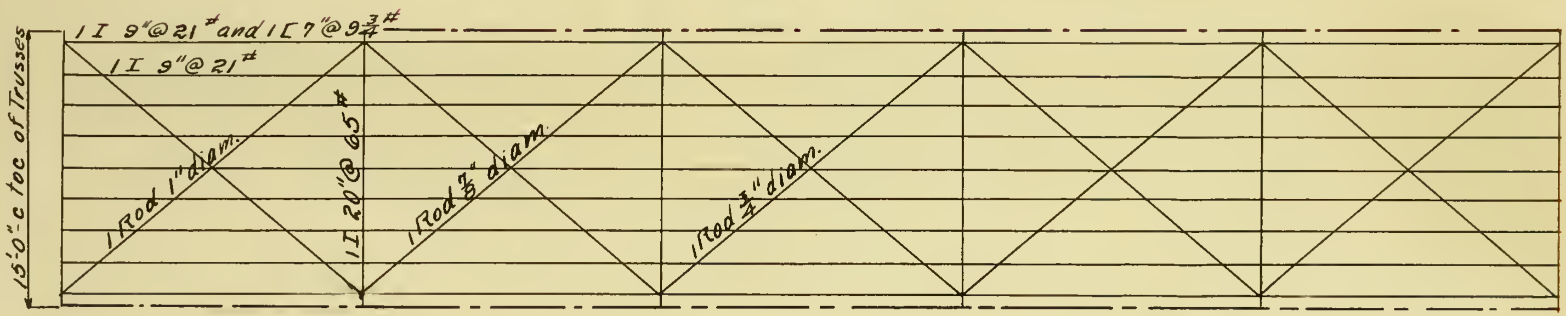
This formula is specified by the Osborn Engineering Company for both railway and highway bridges.

Comparatively few tests have been made on highway bridges; and it is the purpose of this thesis to determine by tests the actual stresses in various members of a highway bridge, and to compare these with calculated stresses. The apparatus used is the same as that employed in the American Railway Engineering and Maintenance of Way Association tests on railway bridges; and since the loading is light compared with railroad rolling stock, and hence the resulting stresses comparatively small, the inaccuracy of the apparatus is increased by the vibration of the members to which it is attached, and thus a correspondingly greater percentage of error is produced in these results than was present in the records taken on railway bridges.

The bridge used in making the tests for this thesis is a five-panel through Pratt truss bridge with a span of 78 feet, 0¼ inch, center to center of end pins. The height is 16 feet, the distance center to center of trusses 15 feet, and the width of roadway 14 feet. The floor is of reinforced concrete. It is situated about two miles northeast of Urbana, Illinois, across the Salt Fork of the Illinois River. Plate I is a stress sheet of this bridge, showing the sizes of the members, and the dead panel load used.

PLATE I.





Weight of Trusses and Laterals	19490 Lb.
Weight of Floor Beams	4435 Lb.
Weight of Stringers	14865 Lb.
Weight of Reinforced Concrete Floor,	84110 Lb.
Weight of Earth Cushion on Floor.	26400 Lb.
Total Weight of Bridge	149300 Lb.

Dead Panel Load = 14930 Lb.

II. THEORY OF IMPACT.

------------0------------

Impact may be defined as the sudden application, to a bar or beam, of a load which is in motion before it is applied. A beam or bridge is subject to both dead and live load stresses, the former due to its own weight, and the latter due to the weight of the traffic that passes over it. The flexural stresses are found by the application of the principles of equilibrium of forces, and are usually computed for dead and live loads separately, each regarded as a static load. The live load, however, really produces stresses greater than the computed ones because it is applied more quickly, and hence it is necessary to multiply the computed stresses by a factor called the "coefficient of impact" in order to obtain the increased stresses due to suddenness of application.

It is known from experiments that an instantaneously applied load produces double the stress and double the deformation that is caused by a static load of the same weight. However, a load moving over a bridge is not applied instantaneously, and hence it is probable that the resulting stress lies between that produced by a static load and that produced by an equal load applied suddenly.

Various methods are in use for assuming values for the coefficient of impact, but in all of them no attention is paid to the time in which the stress is produced; and, in fact, they rest upon no theoretical basis except the law that a suddenly applied load produces double the stress of a static load. Some engineers regard the coefficient as unity for all cases of live load, and hence double the stresses due to live load in the designing of members. Many others assume the coefficient to be less than unity, using higher values for light bridges than for heavy ones, while some make the coefficient depend upon the length of span, and take it higher for short spans than for long ones. Some engineers have combined theory with experiments, and have derived formulae for the determination of impact stresses; but since these formulae are all more or less empirical, they are little used in bridge design or investigation except to compare actual stresses with

those computed from them.

The stress produced by impact is larger for great than for small velocities, as is proved by dropping a weight from different heights onto a beam, the weight thus having acquired different velocities before coming into contact with the beam. The impact stress is also dependent on the static stress that may exist in the beam when the moving load is applied.

When a sudden load is applied to a beam, or other member of a bridge, a series of oscillations ensues which increases from zero to a maximum when the full weight of the load is on the beam, and then gradually decreases to zero again as the bar comes to rest. These vibrations produce variations in the stress in the extreme fibres of the beam; and although this variation may remain below the elastic limit of the material, yet if alternate stresses occur, and the operation is repeated a sufficiently large number of times, the beam will ultimately fail. From experiments made by Wohler, Baushinger, and others, on steel under repeated stress, the following laws have been deduced.

(1). The rupture of a bar may be caused by repeated applications of a unit stress less than the ultimate strength of the material.

(2). The greater the range of stress, the less is the unit stress required to produce rupture after an enormous number of applications.

(3). When the unit stress in a bar varies from zero up to the elastic limit, an enormous number of applications is required to cause rupture.

(4). A range of stress from tension into compression and back again produces rupture with a less number of applications than the same range in stress of one kind only.

(5). When the range of stress in tension is equal to that in compression, the unit stress that produces rupture after an enormous

number of applications is a little greater than one half of the elastic limit.

The term "enormous number" means about forty millions, that being roughly the number used in the experiments above mentioned to cause the rupture of a bar. For all cases of repeated stress in bridges, this great number will not be exceeded during the natural life of the structure, and therefore the above laws do not apply.

III. THEORETICAL STRESSES AND DEFLECTIONS.

---------------0---------------

The theoretical stresses shown in the following tables are computed according to the standard method of obtaining stresses in the members of a Pratt truss. The weight of the bridge used in computing the dead load stresses was calculated from blue prints of the bridge made by the Attica Bridge Co., of Attica, Indiana. In obtaining the stresses in the joists due to live load, the weight of the team and wagon was considered as distributed over two joists, while the weight of the fifteen-ton engine, and of the eight-ton roller was considered as distribut- over four joists. Whether or not this is the true distribution of the load can be determined only by experiment. The deflection of the bridge under the weight of the lumber wagon, and of the eight-ton roller was determined theoretically by the method given in Part II of Merriman and Jacoby's text book on Roofs and Bridges. In each case the load was placed in a position to cause the maximum shear in the middle panel of the bridge.

The dead load stresses are given in each of the following tables. Table I shows the stresses due to the fifteen-ton engine of Waddell's Specifications. The dimensions of this engine are shown in Fig. 1. The impact stress is computed from each of the three formulae given in the introduction, and the maximum impact stress is combined with the dead load stress and the static live load stress to produce the maximum stress given in the last column of the table.

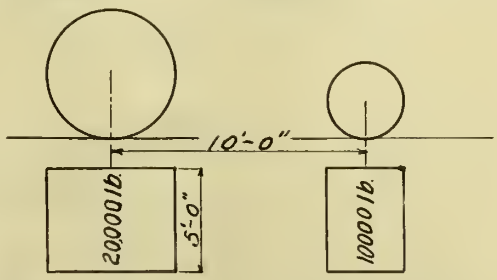


Fig 1. Waddell's 15-Ton Engine.

The maximum shear in each panel is produced when the rear wheel of the engine is at the panel point, with the front wheel in the panel, and the maximum stresses are computed from these shears.

The wagon used in the tests is shown in Fig. 2. Lumber was used for loading the wagon. The stresses due to the weight and impact of this wagon are given in Tables II, III, and IV, each of these tables showing impact stresses as computed from one of the three formulae given in the introduction. The maximum

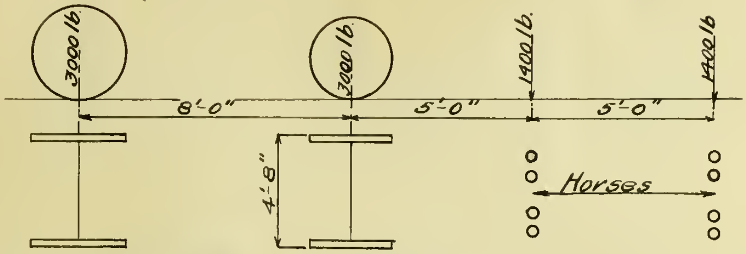


Fig. 2. 8,800-Lb. Lumber Wagon.

shear in each panel is produced when the front wheels of the wagon are at the panel point, with the horses in the panel, and the theoretical stresses are computed from these shears.

The eight-ton road roller used in the tests is owned by R. A. Stipes, of Champaign, Illinois, and is the Universal Road Roller No. 8012, made by the Julian Scholl Co., of New York. The roller is shown in Fig. 3. The stresses due to the

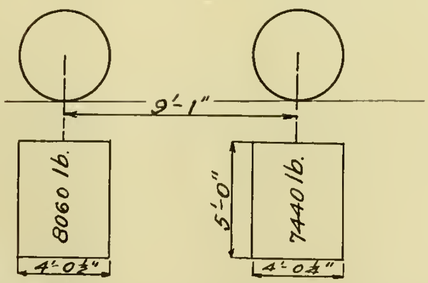


Fig 3. Universal Road Roller.

weight and impact of the roller are given in Tables V, VI, and VII, each of these tables showing theoretical impact stresses as computed from one of the three formulae given in the introduction. The maximum shear in each panel is produced when the rear wheel of the roller is at the panel point, with the front wheel in the panel, and the theoretical stresses are computed with the roller in this position.

Table eight is a summary of the stresses given in Tables I to VII.

TABLE I

STRESSES IN TRUSS

Loaded with Waddell's 15-Ton Engine.

Member	STRESSES						Remarks
	Dead Load	Static Live Load	IMPACT			Total Stress	
			$I=S(\frac{S}{S+D})$	$I=S(\frac{a}{L+b})$	$I=S(\frac{a}{L+b})$	Maximum	
L_0U_1	−41.80	−8.22	−1.35	−4.68	−3.79	−54.70	Second Impact Column,
U_1U_2	−43.70	−8.62	−1.42	−4.75	−3.90	−57.07	a = 100 feet. b = 150 feet.
U_2U_3	−43.70	−8.62	−1.42	−4.75	−3.90	−57.07	Third Impact Column
L_0L_2	+29.20	+9.20	+2.20	+5.52	+4.40	+43.92	a = 150 feet. b = 300 feet.
L_2L_3	+43.70	+8.62	+1.42	+4.75	+3.90	+57.07	S = static live load stress.
U_1L_1	+14.93	+11.80	+5.21	+6.72	+5.44	+33.45	L = length of span loaded to pro-
U_2L_2	0	−2.95	−2.95	−1.43	−1.24	−5.90	duce maximum stress.
U_1L_2	+20.90	+5.50	+1.15	+2.88	+2.42	+29.28	D = dead load stress.
U_2L_3	0	+4.12	+4.12	+2.27	+1.86	+8.24	
U_3L_2	0	+4.12	+4.12	+2.27	+1.86	+8.24	
Beam	±5.60	±7.89	±4.61	±4.92	±3.81	±18.41	
Joist	±5.55	±11.80	±8.04	±7.38	±5.72	±25.39.	

TABLE II

STRESSES IN TRUSS

Loaded with 8800 lb. Wagon

Member	STRESSES.							Remarks
	Dead Load	Live Load Theoretical.	Impact. $I = S\left(\frac{S}{S+D}\right)$	Total Stress Theoretical.	Static Stress Experimental.	Impact Stress Experimental	Total Stress Experimental.	
L_0U_1	-41.80	-3.67	-.296	-45.77	-0.200	-0.200	-42.20	In Impact Column,
U_1U_2	-43.70	-4.48	-.500	-48.68	-0.450	-0.350	-44.50	S = Theoretical live load stress.
U_2U_3	-43.70	-4.48	-.500	-48.68	-0.600	-0.450	-44.75	D = Dead load stress.
L_0L_2	+29.20	+2.76	+.239	+32.20	+0.400	+0.900	+30.50	
L_2L_3	+43.70	+4.48	+.500	+48.68				
U_1L_1	+14.93	+5.89	+1.66	+22.48	+1.000	+0.900	+16.83	
U_2L_2	0	-1.02	-1.02	-2.04	-0.300	-0.100	-0.400	
U_1L_2	+20.90	+2.62	+.292	+22.81	+1.600	+1.500	+24.00	
U_2L_3	0	+1.42	+1.42	+2.84	+2.000	+4.000	+6.000	
U_3L_2	0	+1.42	+1.42	+2.84	+2.000	+4.000	+6.000	
Beam	±5.60	±2.59	±.818	±9.01				
Joist.	±5.55	±2.16	±.818	±8.63	±0.550	±0.700	±6.80	

TABLE III

STRESSES IN TRUSS

Loaded with 8800 lb. Wagon.

Member.	STRESSES							Remarks.
	Dead Load	Live Load Theoretical	Impact $I = S(\frac{a}{L+b})$	Total Stress Theoretical	Static Stress Experimental	Impact Stress Experimental	Total Stress Experimental.	
L_0U_1	-41.80	-3.67	-2.09	-47.56	-0.200	-0.200	-42.20	In Impact Column.
U_1U_2	-43.70	-4.48	-2.37	-50.55	-0.450	-0.350	-44.50	a = 100 feet, b = 150 feet.
U_2U_3	-43.70	-4.48	-2.37	-50.55	-0.600	-0.450	-44.75	S = Theoretical live-load Stress.
L_0L_2	+29.20	+2.76	+1.59	+33.55	+0.400	+0.900	+30.50	L = Length of span loaded to
L_2L_3	+43.70	+4.48	+2.37	+50.55				produce maximum stress.
U_1L_1	+14.93	+5.89	+3.36	+24.18	+1.000	+0.900	+16.83	
U_2L_2	0	-1.02	-.49	-1.51	-0.300	-0.100	-0.400	
U_1L_2	+20.90	+2.62	+1.37	+3.99	+1.600	+1.500	+24.00	
U_2L_3	0	+1.42	+1.38	+2.80	+2.000	+4.000	+6.000	
U_3L_2	0	+1.42	+1.38	+2.80	+2.000	+4.000	+6.000	
Beam	±5.60	±2.59	±1.59	±9.68				
Joist	±5.55	±2.16	±1.31	±9.02	±0.550	±0.700	±6.80	

TABLE IV.

STRESSES IN TRUSS

Loaded with 8800 lb. Wagon.

Member.	STRESSES Dead Load	Live Load Theoretical	Impact $I = S(\frac{a}{L+b})$	Total Stress Theoretical	Static Stress Experimental	Impact Stress Experimental.	Total Stress Experimental	Remarks
L_0U_1	-41.80	-3.67	-1.69	-47.16	-0.200	-0.200	-42.20	In Impact Column,
U_1U_2	-43.70	-4.48	-1.99	-50.17	-0.450	-0.350	-44.50	a = 150 feet b = 300 feet
U_2U_3	-43.70	-4.48	-1.99	-50.17	-0.600	-0.450	-44.75	S = Theoretical live load stress
L_0L_2	+29.20	+2.76	+1.28	+33.24	+0.400	+0.900	+30.50	L = length of span loaded
L_2L_3	+43.70	+4.48	+1.99	+50.17				to produce maximum stress
U_1L_1	+14.93	+5.89	+2.71	+23.53	+1.000	+0.900	+16.83	
U_2L_2	0	-1.02	-.43	-1.45	-0.300	-0.100	-0.400	
U_1L_2	+20.90	+2.62	+1.15	+24.67	+1.600	+1.500	+24.00	
U_2L_3	0	+1.42	+.63	+2.05	+2.000	+4.000	+6.000	
U_3L_2	0	+1.42	+.63	+2.05	+2.000	+4.000	+6.000	
Beam	±5.60	±2.59	±1.24	±9.43				
Joist	±5.55	±2.16	±1.03	±8.84	±0.550	±0.700	±6.80	

TABLE V

STRESSES IN TRUSS

Loaded with 8-ton Roller.

Member	STRESSES							Remarks.
	Dead Load	Live Load Theoretical	Impact $I = S(\frac{S}{S+D})$	Total Stress Theoretical	Static Stress Experimental	Impact Stress Experimental	Total Stress Experimental	
L_oU_1	-41.80	-3.89	-.331	-46.02	-0.800	29.5 -0.250	-42.85	In Impact Column,
U_1U_2	-43.70	-4.06	-.345	-48.10	-1.500	20 -0.300	-45.50	S = Theoretical live load stress
U_2U_3	-43.70	-4.06	-.345	-48.10	-1.400	50 -0.700	-45.80	D = Dead load stress
L_oL_2	+29.20	+4.34	+.561	+34.10	+1.800	50 +0.900	+31.90	
L_2L_3	+43.70	+4.06	+.345	+48.10	+0.950	42 +0.400	+45.05	
U_1L_1	+14.93	+5.57	+1.51	+22.01	+2.400	25 +1.000	+18.33	
U_2L_2	0	-1.39	-1.39	-2.78	-0.250	40 -0.100	-0.350	
U_1L_2	+20.90	+2.58	+.283	+23.76	+3.000	50 +1.500	+25.40	
U_2L_3	0	+1.94	+1.94	+3.88	+3.000	+3.000	+6.00	
U_3L_2	0	+1.94	+1.94	+3.88	+3.000	+3.000	+6.00	
Beam.	±5.60	±3.81	±1.54	±10.95	±1.400	±0.100	±7.10	
Joist.	±5.55	±5.00	±2.37	±12.92	±2.200		±7.75	

TABLE VI

STRESSES IN TRUSS

Loaded with 8-ton Roller

Member.	STRESSES							Remarks
	Dead Load	Live Load Theoretical	Impact. $I = S(\frac{a}{L+b})$	Total Stress Theoretical	Static Stress Experimental	Impact Stress Experimental	Total Stress Experimental	
L_0U_1	-41.80	-3.89	-2.20	-47.89	-0.800	-0.250	-42.85	In Impact Column
U_1U_2	-43.70	-4.06	-2.24	-50.00	-1.500	-0.300	-45.50	a = 100 feet b = 150 ft
U_2U_3	-43.70	-4.06	-2.24	-50.00	-1.400	-0.700	-45.80	S = Theoretical live load stress
L_0L_2	+29.20	+4.34	+2.62	+36.16	+1.800	+0.900	+31.90	L = Length of span loaded to
L_2L_3	+43.70	+4.06	+2.24	+50.00	+0.950	+0.400	+45.05	produce maximum stress.
U_1L_1	+14.93	+5.57	+3.19	+23.69	+2.400	+1.000	+18.30	
U_2L_2	0	-1.39	-.90	-2.29	-0.250	-0.100	-0.350	
U_1L_2	+20.90	+2.58	+1.36	+24.84	+3.000	+3.000	+25.40	
U_2L_3	0	+1.94	+1.07	+3.01	+3.000	+1.500	+6.00	
U_3L_2	0	+1.94	+1.07	+3.01	+3.000	+3.000	+6.00	
Beam	±5.60	±3.81	±2.35	±11.76	±1.400	±0.100	±7.10	
Joist.	±5.55	±5.00	±3.13	±13.68	±2.200		±7.75	

TABLE VII.

STRESSES IN TRUSS

Loaded with 8-ton Roller.

Member	STRESSES							Remarks.
	Dead Load	Live Load Theoretical	Impact $I=S\left(\frac{a}{L+b}\right)$	Total Stress Theoretical	Static Stress Experimental	Impact Stress Experimental	Total Stress Experimental	
L_0U_1	-41.80	-3.89	-1.80	-47.49	-0.800	-0.250	-42.85	In Impact Column
U_1U_2	-43.70	-4.06	-1.84	-49.60	-1.500	-0.300	-45.50	a=150 feet. b=300 feet.
U_2U_3	-43.70	-4.06	-1.84	-49.60	-1.400	-0.700	-45.80	S=Theoretical live load stress
L_0L_2	+29.20	+4.34	+2.06	+35.60	+1.800	+0.900	+31.90	L=Length of span loaded to
L_2L_3	+43.70	+4.06	+1.84	+49.60	+0.950	+0.400	+45.05	produce maximum stress.
U_1L_1	+14.93	+5.57	+2.57	+23.07	+2.400	+1.000	+18.33	
U_2L_2	0.0	-1.39	-.59	-1.98	-0.250	-0.100	-0.350	
U_1L_2	+20.90	+2.58	+1.14	+24.62	+3.000	+1.500	+25.40	
U_2L_3	0.0	+1.94	+.88	+2.82	+3.000	+3.000	+6.00	
U_3L_2	0.0	+1.94	+.88	+2.82	+3.000	+3.000	+6.00	
Beam	±5.60	±3.81	±1.83	±11.24	±1.400	±0.100	±7.10	
Joist	±5.55	±5.00	±2.42	±12.97	±2.200		±7.75	

TABLE VIII

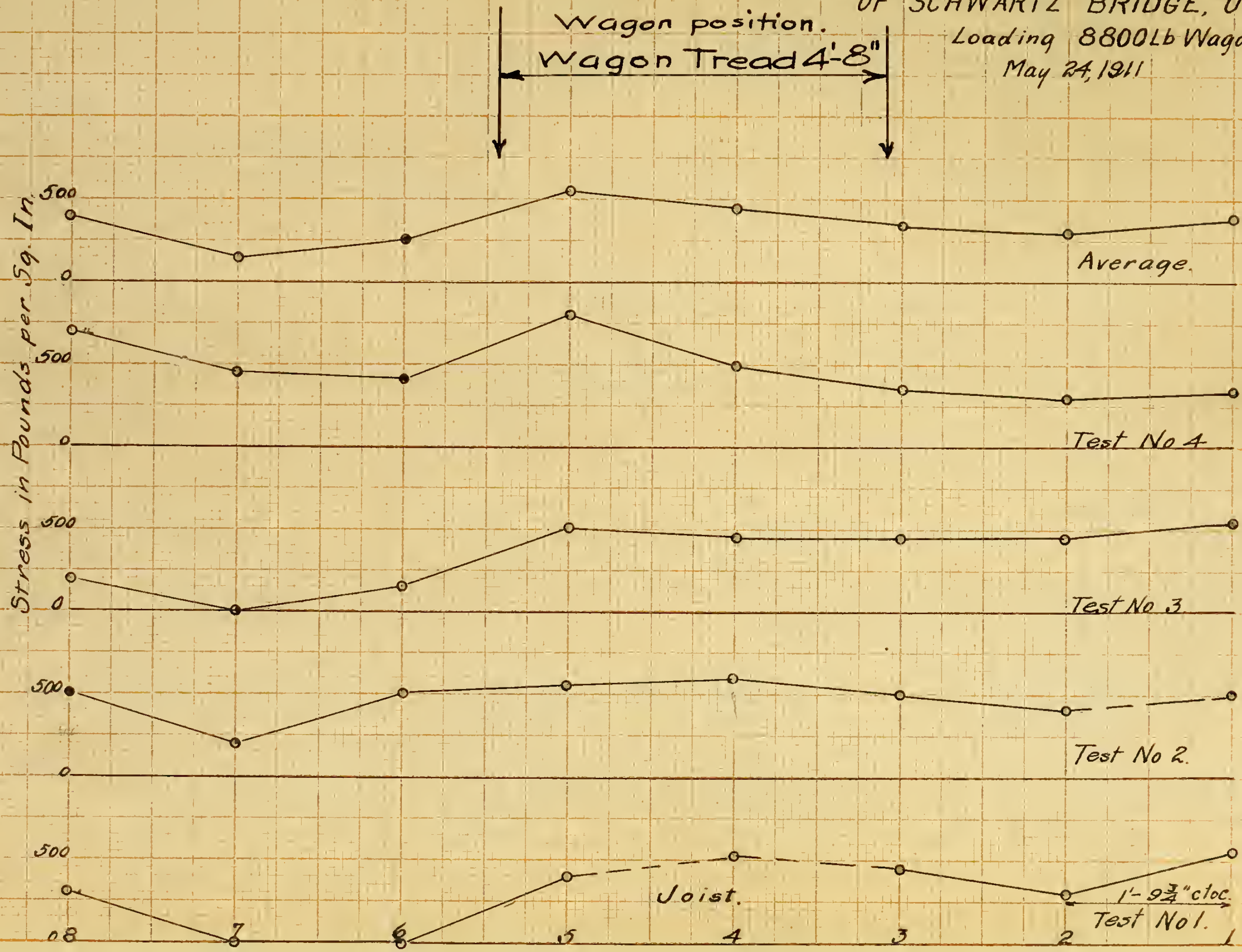
SUMMARY OF STRESSES

Member.	Dead-Load Stress	THEORETICAL LIVE LOAD STRESS			EXP. STATIC STRESS		EXPERIMENTAL IMPACT STRESS			TOTAL STRESS		
		15-Ton Engine	8800 lb. Wagon	8-Ton Roller	8800 lb. Wagon.	8-Ton Roller.	15-Ton Engine*	8800 lb. Wagon	8-Ton Roller.	15-Ton Engine	8800 lb Wagon	8-Ton Roller.
L_0U_1	-41.80	-8.22	-3.67	-3.89	-0.200	-0.800	-4.68	-0.200	-0.250	-54.70	-42.20	-42.85
U_1U_2	-43.70	-8.62	-4.48	-4.06	-0.450	-1.500	-4.75	-0.350	-0.300	-57.07	-44.50	-45.50
U_2U_3	-43.70	-8.62	-4.48	-4.06	-0.600	-1.400	-4.75	-0.450	-0.700	-57.07	-44.75	-45.80
L_0L_2	+29.20	+9.20	+2.76	+4.34	+0.400	+1.800	+5.52	+0.900	+0.900	+43.92	+30.50	+31.90
L_2L_3	+43.70	+8.62	+4.48	+4.06		+0.950	+4.75		+0.400	+57.07		+45.05
U_1L_1	+14.93	+11.80	+5.89	+5.57	+1.000	+2.400	+6.72	+0.900	+1.000	+33.45	+16.83	+18.33
U_2L_2	0	-2.95	-1.02	-1.39	-0.300	-0.250	-1.43	-0.100	-0.100	-4.38	-0.400	-0.350
U_1L_2	+20.90	+5.50	+2.62	+2.58	+1.600	+3.000	+2.88	+1.500	+1.500	+29.28	+24.00	+25.40
U_2L_3	0	+4.12	+1.42	+1.94	+2.000	+3.000	+4.12	+4.000	+3.000	+8.24	+6.00	+6.00
U_3L_2	0	+4.12	+1.42	+1.94	+2.000	+3.000	+4.12	+4.000	+3.000	+8.24	+6.00	+6.00
Beam.	±5.60	±7.89	±2.59	±3.81		±1.400	±4.92		±0.100	±18.31		±7.100
Joist.	±5.55	±11.80	±2.16	±5.00	±0.550	±2.200	±7.38	±0.700		±24.73	±6.80	±7.750

* Theoretical.

PLATE II.

GRAPHS
SHOWING DISTRIBUTION OF STRESSES
IN JOISTS OF REINFORCED CONCRETE FLOOR
OF SCHWARTZ BRIDGE, URBANA ILL.
Loading 8800 Lb Wagon.
May 24, 1911



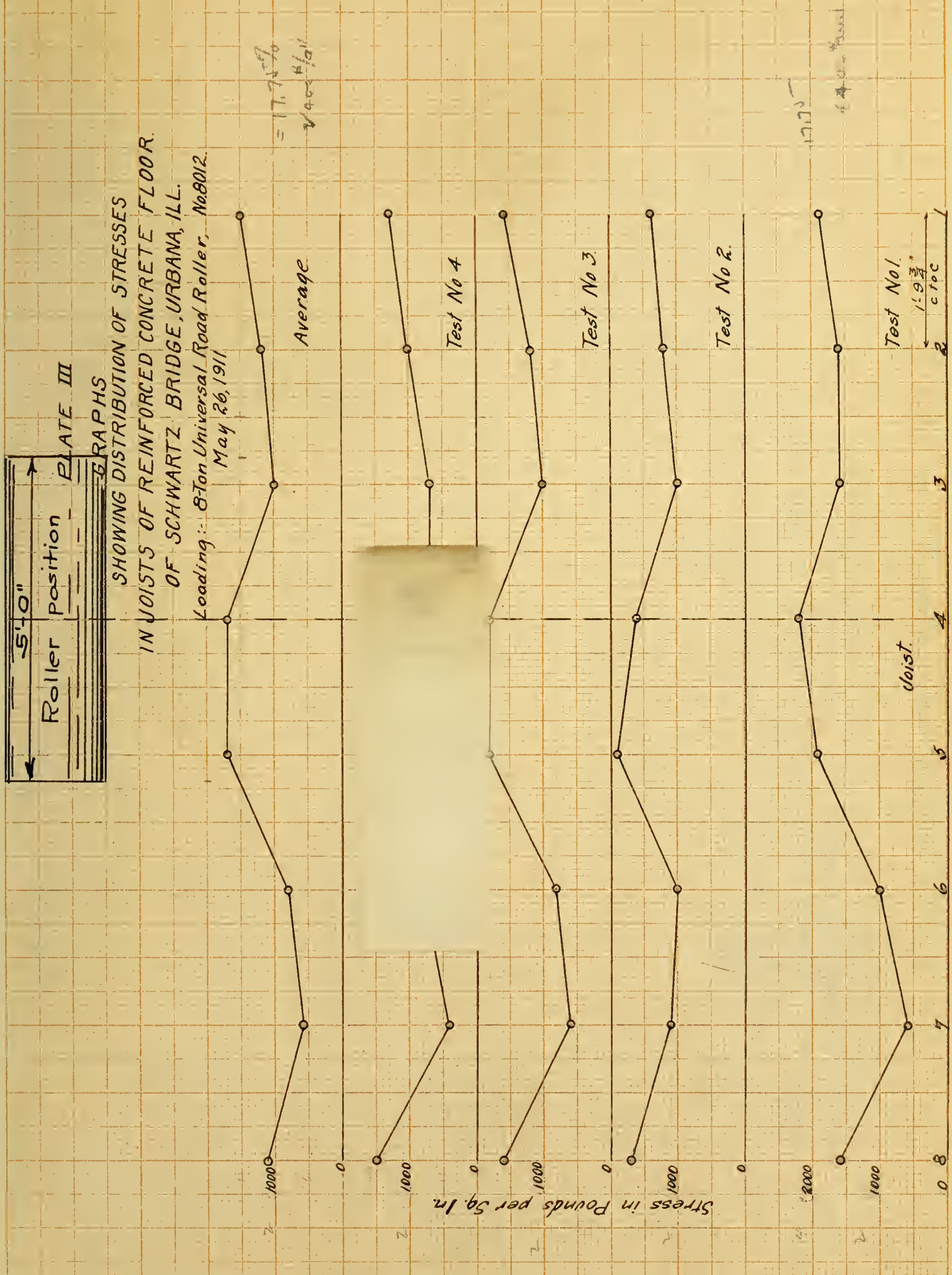
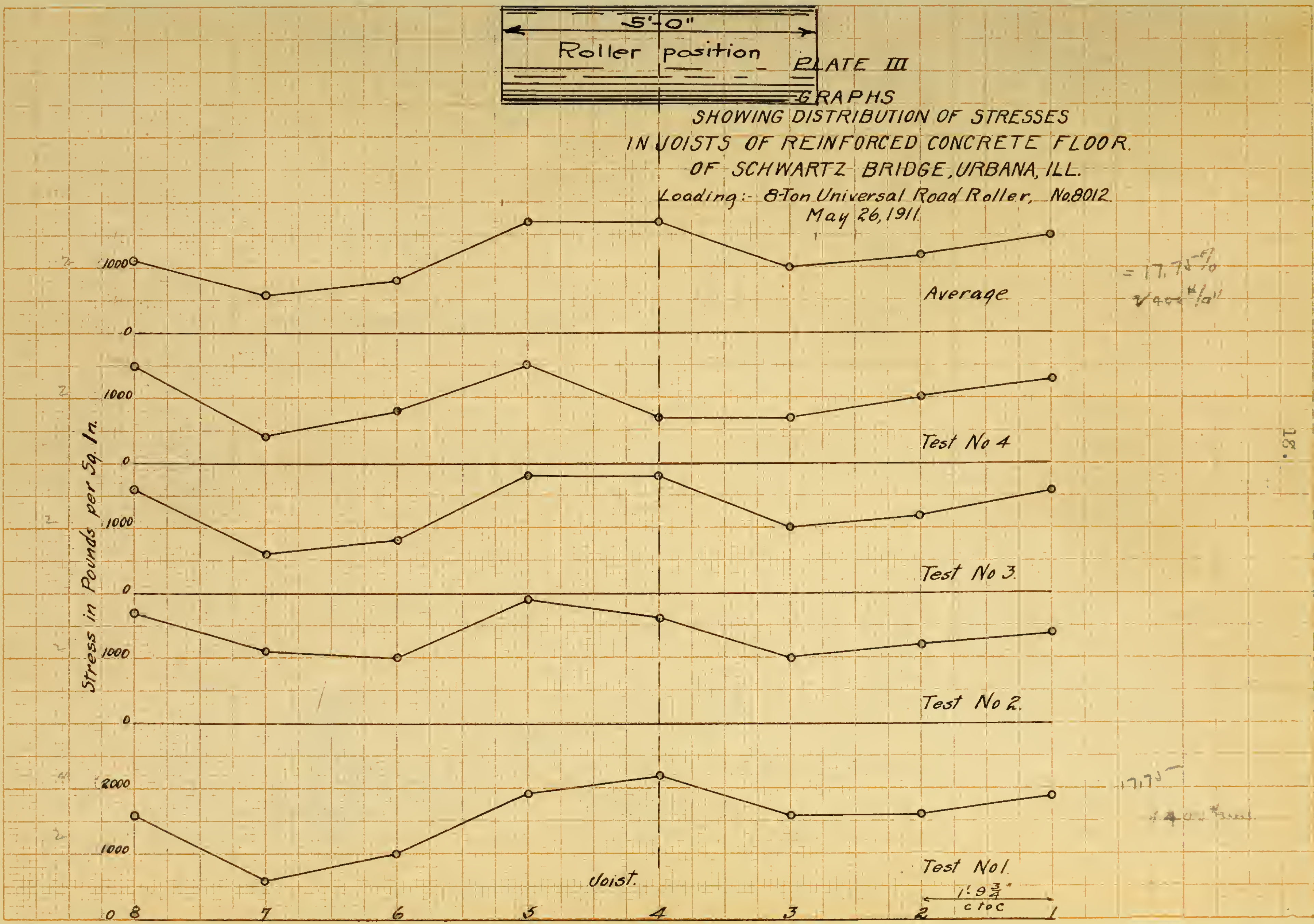


PLATE III
GRAPHS
SHOWING DISTRIBUTION OF STRESSES
IN JOISTS OF REINFORCED CONCRETE FLOOR
OF SCHWARTZ BRIDGE, URBANA, ILL.
Loading: 8-Ton Universal Road Roller, No.8012.
May 26, 1911.
Roller position
5'-0"
Average
Test No 4
Test No 3
Test No 2
Test No 1.
1'-9 3/4" c to c
Joist.
1
2
3
4
5
6
7
8
Stress in Pounds per Sq. In.
1000
0
1000
0
1000
0
1000
0
2000
1000
0


5'-0"
Roller position
PLATE III
GRAPHS
SHOWING DISTRIBUTION OF STRESSES
IN JOISTS OF REINFORCED CONCRETE FLOOR.
OF SCHWARTZ BRIDGE, URBANA, ILL.
Loading:- 8-Ton Universal Road Roller, No.8012.
May 26, 1911.
Average
Test No 4
Test No 3
Test No 2.
Test No1
Stress in Pounds per Sq. In.
1000
0
2000
Joist.
8
7
6
5
4
3
2
1
1'-9 3/4"
c to c

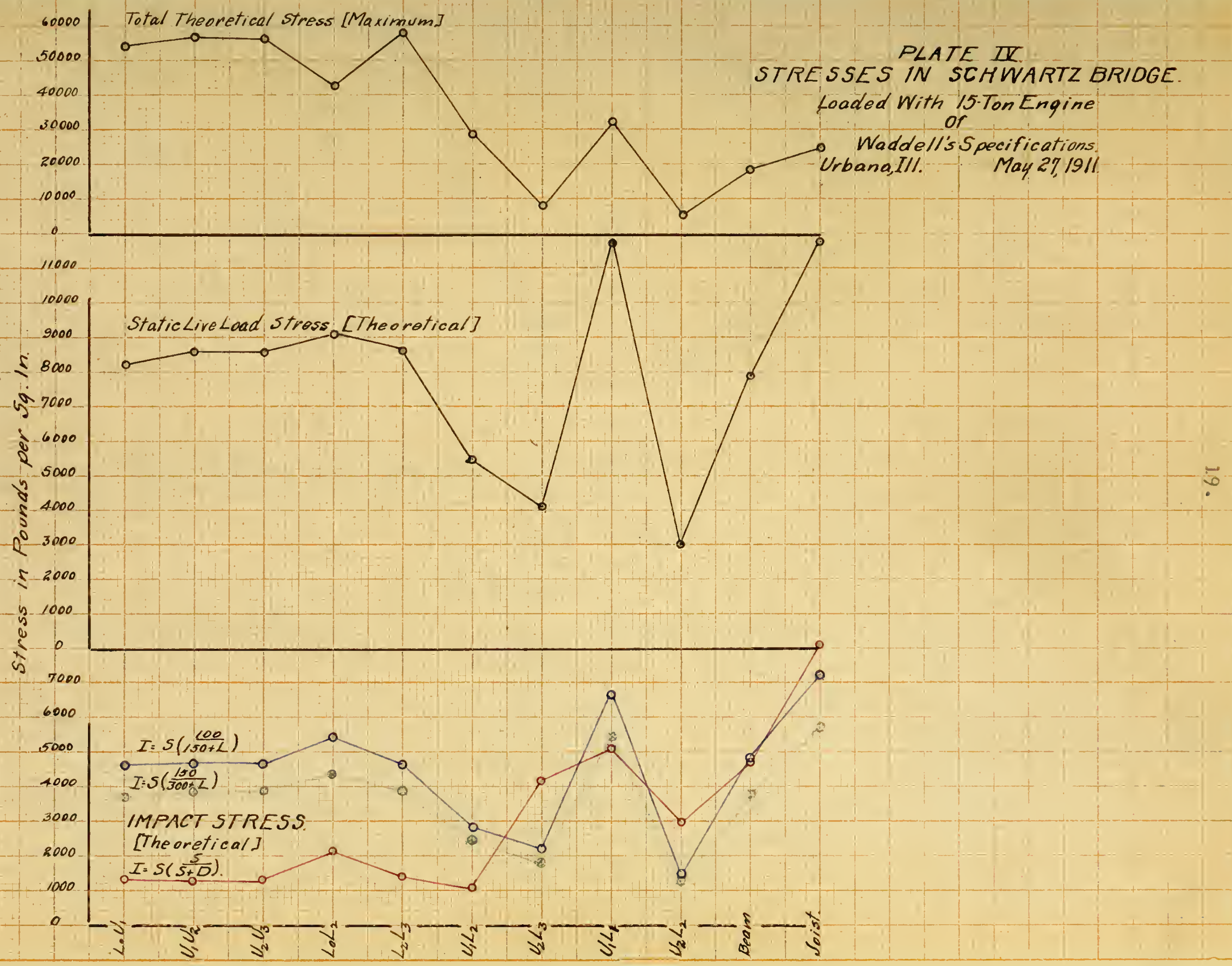


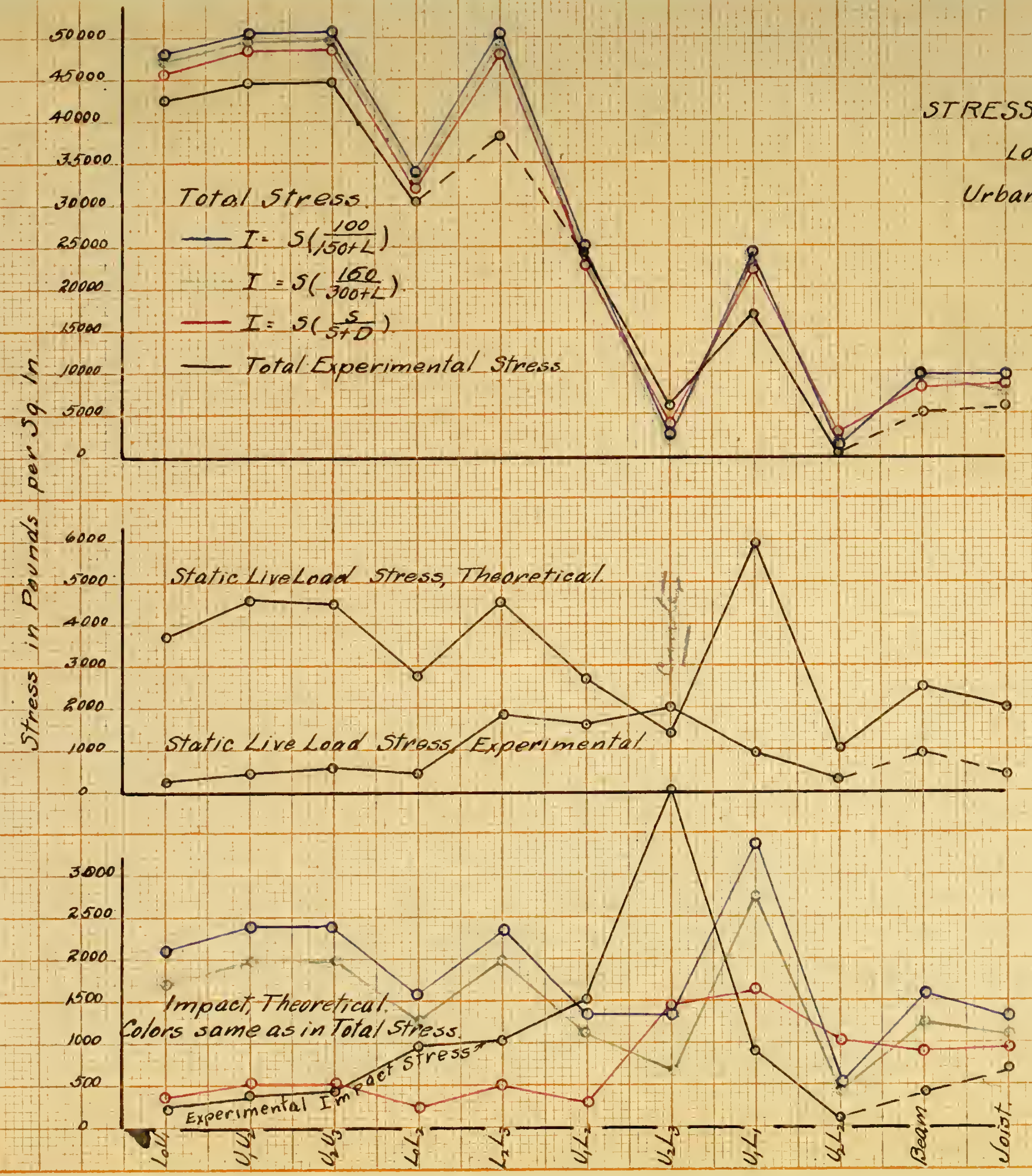
PLATE IV.
STRESSES IN SCHWARTZ BRIDGE.
Loaded With 15-Ton Engine
of
Waddell's Specifications.
Urbana, Ill. May 27, 1911.
Total Theoretical Stress [Maximum]
Static Live Load Stress [Theoretical]
IMPACT STRESS.
[Theoretical]
$I = S(\frac{100}{150+L})$
$I = S(\frac{150}{300+L})$
$I = S(\frac{S}{S+D})$.
Stress in Pounds per Sq. In.
60000
50000
40000
30000
20000
10000
0
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
7000
6000
5000
4000
3000
2000
1000
0
L_0U_1
U_1U_2
U_2U_3
L_0L_2
L_2L_3
U_1L_2
U_2L_3
U_1L_1
U_2L_2
Beam
Joist.

PLATE V

STRESSES IN SCHWARTZ BRIDGE.

Loaded With 8800-Lb Wagon

Urbana, Ill. May 24, 1911.



U OF I. B. S. FORM 3

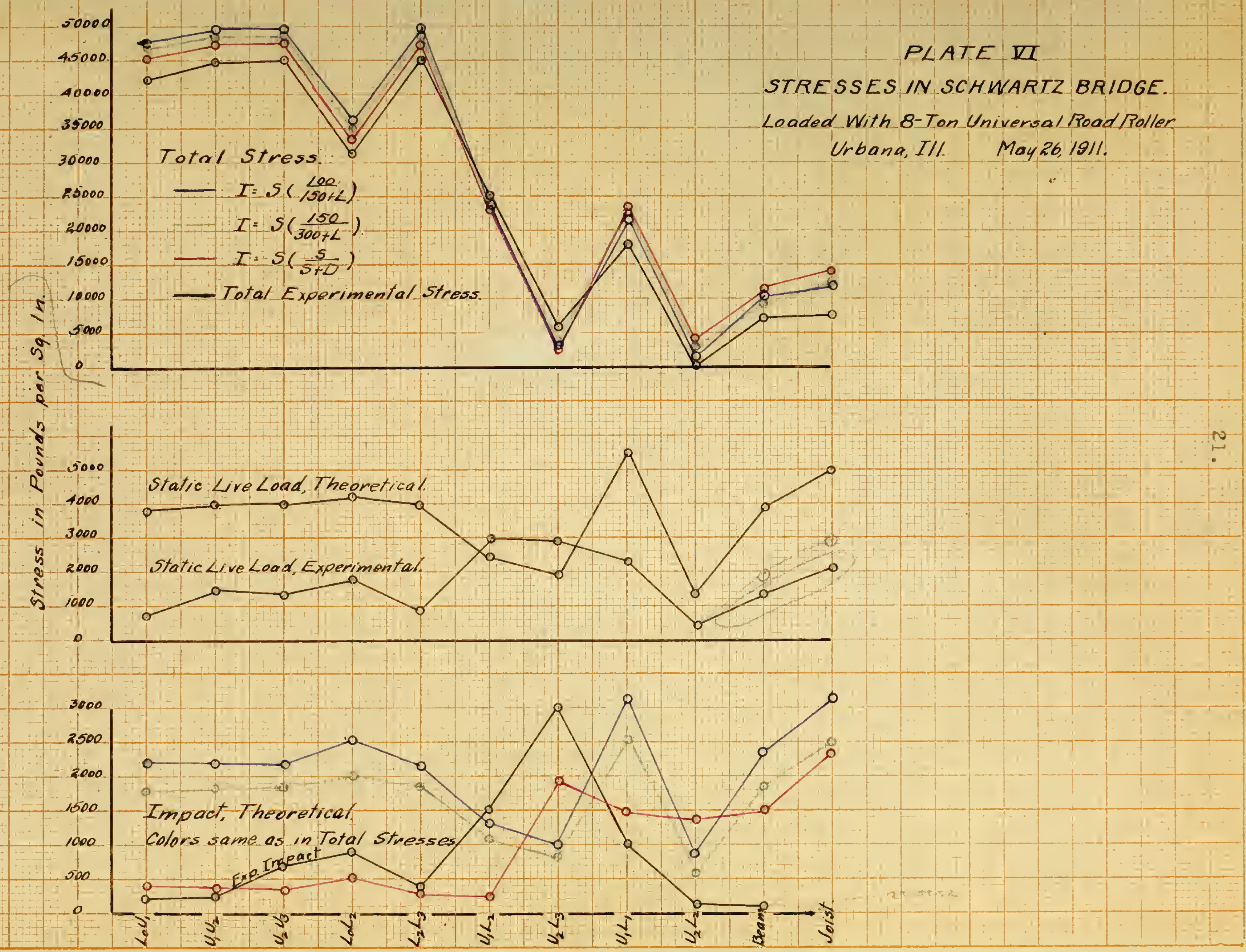


PLATE VI
STRESSES IN SCHWARTZ BRIDGE.
Loaded With 8-Ton Universal Road Roller
Urbana, Ill. May 26, 1911.
Total Stress.
$I = S\left(\frac{L00}{150+L}\right)$
$I = S\left(\frac{150}{300+L}\right)$
$I = S\left(\frac{S}{S+D}\right)$
Total Experimental Stress.
Static Live Load, Theoretical.
Static Live Load, Experimental.
Impact, Theoretical.
Colors same as in Total Stresses.
Exp. Impact
Stress in Pounds per Sq. In.
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
5000
4000
3000
2000
1000
0
3000
2500
2000
1500
1000
500
0
L_0U_1
U_1U_2
U_2U_3
L_0L_2
L_2L_3
U_1L_2
U_2L_3
U_1L_1
U_2L_2
Beam
Joist

V. COMPARISONS.

----------0----------

The actual stresses in the various members of the bridge do not agree at all with the theoretical stresses. This fact is probably due to the unusually rigid floor system. The reinforced concrete floor tends to distribute the load in such a way that the actual stress in any member is quite small.

The impact stresses do not follow a fixed rule, but rather the percentage of impact seems to be greatest in the smallest members. During the tests with the eight-ton roller the greatest impact stress was recorded when the roller first entered the bridge, or when it struck an uneven place in the floor cushion, causing a sidewise rocking motion of the roller. This condition is similar to uneven rail joints on a railway bridge, which is known to be an important factor in the determination of impact due to the passage of trains. In a few cases the impact was about half the static stress, and in some cases the static stress and the impact stress were equal. In Table V, the ratio of impact to static stress in the counters of the middle panel agree well with the formula given for impact in that table. The formula given in this table gives stresses closer to the experimental results than either of the other formulae.

The tests for distribution of the load over the floor joists are shown graphically in Plates II and III. The high stress in the outside joist is probably due to the fact that this joist is tied to the third joist from the outside by a steel rod through the concrete, which carries part of the load from the latter joist to that on the outside. The joist directly under the load takes about twenty per cent of the total load. The floor beam showed an actual static stress equal to about one half the computed stress.

The deflection of the bridge at the center due to the weight of the roller as computed from theory was 0.0903 inch, and the actual deflection as observed in the experiments was 0.110 inch. The deflection due to the weight of the wagon was computed to be 0.0405 inch, and the experimental deflection was 0.050 inch. This

seems to indicate that the total deflection is little influenced by the rigidity of the floor, and that the theory as given in Part II of Merriman and Jacoby's text book on Roofs and Bridges is accurate.

Plates IV, V, and VI show graphically the theoretical and experimental stresses, each plate showing the stresses due to one of the three loadings, and theoretical impact stresses according to each of the three formulae given in the introduction.

VI. CONCLUSIONS.

----------0----------

The bridge tested is of an unusually rigid type. The tests show stresses which are much lower than the computed stresses, in all of the members with the exception of the counters in the middle panel where the experimental stresses are larger than the computed. This is due to the fact that the reinforced concrete floor, acting as a continuous beam, carries the load to all of the panel points, distributing it more uniformly than would be the case with other types of floors. From this it is concluded that the truss members of this bridge are unnecessarily large, and that considerable economy could have been effected by considering the load as distributed by the floor. A portion of the load is also, in all probability, carried directly to the abutments by the floor.

The tests show that one fifth of the total load is taken by one joist. This is due to the rigidity of the floor, and does not furnish any definite rule for calculating the stresses in the joists of bridges with other types of floors.

In testing with the wagon, tests were made with the horses walking and trotting. In every case the impact stresses with the horses trotting were much higher than those with the horses walking. The impact stresses due to the roller were very little higher than those due to the wagon when the horses were trotting. This seems to indicate that the weight of the load has not so much to do with the impact stress as has the pounding of the horses hoofs and the speed of the load in passing over the bridge. It also shows the advisability of compelling teams to slow down to a walk when crossing a bridge. This is undoubtedly of greater importance with bridges with wooden floors than in the case under consideration. The greatest impact stress due to the roller was caused as the roller entered or left the bridge. At both ends there was a slight unevenness in the earth cushion, which caused the roller to lunge from side to side and bounce slightly. If the floor had been smoth there would have been very little impact. This shows the necessity of keeping the floors in good condition.

UNIVERSITY OF ILLINOIS-URBANA
3 0112 082197184